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PROBABILISTIC TEST PLANNING

J. H. Wiggins Company
1650 South Pacific Coast Highway
Redondo Beach, California 90277

18 July 1977

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A study was conducted to evaluate the failure probabilities for the tunnels to be tested in DIABLO HAWK. The results indicate that 5 - 6 tunnels will be severely damaged, and the test can be improved upon by the addition of one more cylinder. The effects of correlation (among the various structures, concrete, etc.) was simulated computationally using a Monte Carlo procedure. Details of the simulation procedures and the decision theory used are contained in the main text.

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PREFACE

This study was sponsored by the Strategic Structures Division of DNA with Dr. Kent Goering as Technical Monitor; his support and constructive suggestions are gratefully acknowledged. Dr. Jon Collins and Bruce Kennedy of the J.H. Wiggins Company were very instrumental in the success of the project, particularly in the decision analysis, the developing and verifying of the computer program and in providing a constructive review of the report.

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TABLE OF CONTENTS

1. SUMMARY AND INTRODUCTION

1.1 SUMMARY

A study was performed to evaluate failure probabilities for the cylindrical tunnels in the DIABLO HAWK test, to recommend test changes, and to investigate the effects of correlation on the failure probabilities. It is anticipated that either 5 or 6 tunnels will fail in the DIABLO HAWK test, with 5 tunnels experiencing severe damage. A brief decision analysis indicates that the test would be optimized with the addition of one structure designed and situated in order to have a failure probability of .5. However, the new economic benefit is small and in the long run, the omission of the additional structure will have very minor influence. If the structural properties are fully-correlated, and if the applied loading (on the several tunnels) is also fully-correlated, then the predicted number of failures for DIABLO HAWK is expected to fall between 3 and 8 with 90% probability. Additional results of the study include the conclusion that fragility curves can readily be developed for hollow spheres, and the elasto-plastic design of spheres is amenable to computer implementation but is somewhat more complicated than tunnel design.

1.2 INTRODUCTION

Previous studies (Reference 1) were reasonably successful in predicting the number of tunnel failures in the MIGHTY EPIC test. Nevertheless, the previous analyses did not address the DIABLO HAWK test, nor did they include the important effect of correlation among the random variables involved.

The designers of the (MIGHTY EPIC/DIABLO HAWK) test structures wanted to investigate the effects of multiple loadings and also the propagation of a ground shock pulse through "pre-shocked" material. Neither of these effects are addressed in the present report. If one neglects these effects (as the present report does) then it appears that the DIABLO HAWK test is less-than-optimally designed. Suggestions are made herein to improve the probability of obtaining useful data, without altering the effects of multiple loading or pre-shocking.

The two prime sources of correlation for the tunnel structures are

- The commonality of construction procedures, concrete strengths, etc. that may make the structural properties highly correlated, and

- The correlation of weapon output from structure to structure, namely, if the free-field pressure is high, it will be high for all the structures, etc.

The simplest case of full correlation is the easiest to handle computationally. But, in fact, since the rock properties vary throughout the test site, the "fully-correlated" approximation does not represent the real, physical situation. Thus, it is expected that the actual test will lie somewhere between the totally uncorrelated and fully correlated extremes.

From physical arguments, one can determine that the effect of correlating the structural properties and the pressure loading will be to emphasize the extremes of the probability distributions. This intuitive result is borne out by the calculations.

The question of developing fragility curves for hollow spheres is also addressed, as is the problem of elasto-plastic design of spheres in rock. Both of these problems are conceptually straightforward, although the elasto-plastic design is more complicated for spheres than it is for cylinders.

The report includes major sections on predictions and recommendations for DIABLO HAWK, the effect of correlation, sphere calcualtions, and concluding remarks.

2. PROBABILITY OF FAILURE FOR DIABLO HAWK CYLINDERS

2.1 BACKGROUND

In the previous report (Reference 1) probabilistic predictions were made regarding the expected number of failures (for lined tunnels in rock) in the MIGHTY EPIC test. Assuming a factor of $\overset{\times}{\div}$ 1.1 uncertainty in the free-field pressure produced in the tuff, it was stated that "the number of expected failures lies between 4 and 6, within 90 per cent confidence limits." In the test, four (4) tunnels were damaged significantly, with two of the four suffering severe damage and two suffering moderate damage.

The two structures that were severely damaged had a mean probability of failure $\bar{P}_f$ equal to .99, calculated for the factor of 1.1 uncertainty. The other two structures, which received moderate damage, had failure probabilities $\bar{P}_f = .91$ and $\bar{P}_f = .79$, respectively. It is noteworthy that one other structure, which experienced only slight damage, also had a failure probability $\bar{P}_f = .91$.

In the paragraphs which follow, the calculation of similar failure probabilities for the DIABLO HAWK test are outlined. The results show that five (5) tunnels have failure probabilities $\bar{P}_f \geq .97$ for a factor of 1.1 uncertainty in the free-field stress. Consequently, it is predicted that either 5 or 6 tunnels will fail (within 90 per cent confidence limits) in the DIABLO HAWK event. Furthermore, these 5 tunnels are expected to be severly damaged, in a manner like the two MIGHTY EPIC structures which had failure probabilities $\bar{P}_f \geq .99$.

The reader is again cautioned that these predictions do not include the effect of a second loading on the rock medium. If the second loading causes major effects not anticipated at this time, then the predicted number of failures might change significantly.

2.2 EFFECTS OF UNCERTAINTY IN FREE-FIELD PRESSURE

In order to understand the effect that uncertainties in the free-field stress (pressure) have on the mean probability of failure ($\bar{P}_f$), it is worthwhile to consider the steps in the calculation of $\bar{P}_f$. This procedure is outlined in the flow-chart of Figure 2-1. The individual blocks in the flow-chart can be briefly summarized as follows:

Step 1: Calculate the design stress, σ_o, for a particular tunnel. The design stress depends upon the steel thickness, h_s, the concrete thick-

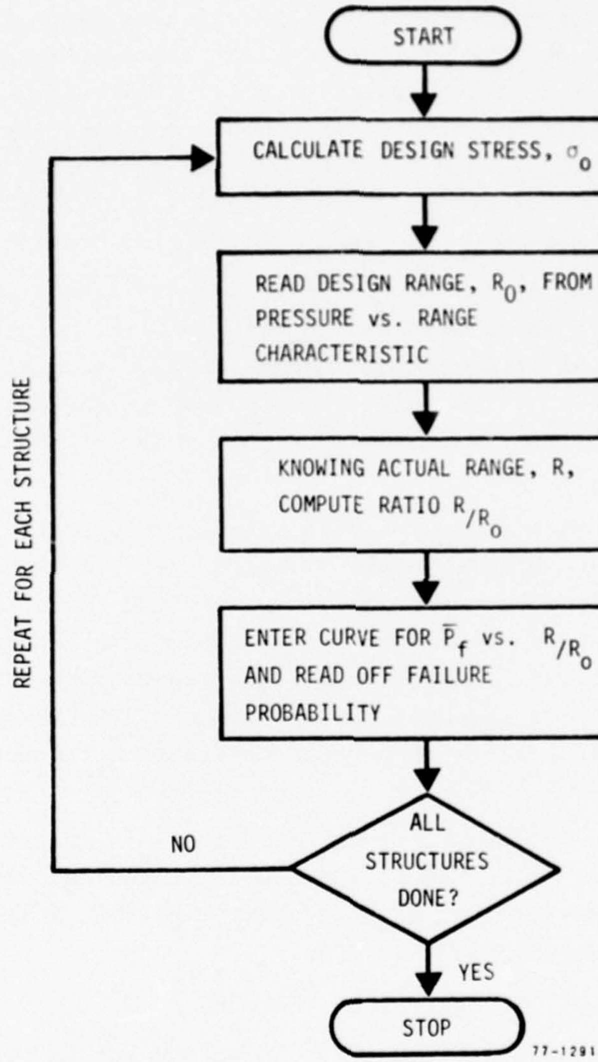

Figure 2-1. Flow of The Calculations

ness, t_c, etc. A failure strain of $\varepsilon_\theta = .05$ was used (cf. Reference 1) along with mean values of the design variables E_{rock}, ν_{rock}, etc. The mean values of these variables are listed in Table 2-1.*

Step 2: Using the design stress, σ_o, and a curve of free-field pressure vs. range (estimated for the event), one can read off a corresponding design range, R_o.

Step 3: From a knowledge of the actual range, R (from the structure to the weapon point) and the design range, R_o, from Step 2, one can compute the dimensionless ratio (R/R_o).

Step 4: Using the curves developed previously, giving $\bar{P}_f$ vs. (R/R_o), one can read off the corresponding mean probability of failure (see Figure 2-2).

Step 5: This procedure is repeated for all the structures involved.

The calculations just outlined were performed for the DIABLO HAWK test cylinders using uncertainty factors of 2 and 4 for the free-field pressure. Similar (but somewhat simpler) calculations were done using an uncertainty factor 1.1. The results of these calculations are given in Table 2-2. As can be seen by referring to the Table, the effect of decreasing the uncertainty in free-field pressure is to decrease the probabilities when $\bar{P}_f \leq .5$ and to increase the probabilities when $\bar{P}_f \geq .5$. This sharpening of the distribution is evident in Figure 2-3, which also shows a suggested "optimum" test design. The reader should be aware that the curves of Figures 2-2 and 2-3 are based upon an assumed value of the "failure strain," namely $\varepsilon_\theta = .05$. Although this value ($\varepsilon_\theta = .05$) gave reasonably good results for the MIGHTY EPIC test, the specification of a failure strain is an involved subject and deserves further study.

The curves of Figure 2-3 substantiate the previous findings (Reference 1, p. 5-6) that "assuming a factor of 4 uncertainty in the free-field pressure, the actual test was reasonably well-designed." Conversely, when the uncertainty is reduced to a factor of 1.1 (which is thought to be representative of the well-calibrated Nevada Test Site) it appears that the combined (MIGHTY EPIC/DIABLO HAWK) test cylinders were less than optimally designed. This finding is reinforced when the probability of exactly N failures is comptuted, as discussed in the following paragraph.

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*These tabulated values were used previously in Reference 1.

Table 2-1. Nominal Values of the Design Variables

DESIGN VARIABLES	NOMINAL VALUE USED
EROCK E_r, YOUNG'S MODULUS FOR ROCK AND GROUT	$.87 \times 10^6$ lb/in^2
GNUROK ν_r, POISSON'S RATIO FOR ROCK AND GROUT	.2
GNUCON ν_c, POISSON'S RATIO FOR CONCRETE	.14
FYIELD f_y, YIELD STRESS FOR THE STEEL LINER	38 kips/in^2
FCPRIM f'_c, UNCONFINED COMPRESSIVE STRENGTH OF CONCRETE	5500 lb/in^2 (static) 8000 lb/in^2 (dynamic)
XKCON k_{sc}, FRICTION-DEPENDENT CONSTANT, FOR CONCRETE	$\mu_k = 3$ $\mu_o = 30°$
XKROK k_{sr}, FRICTION-DEPENDENT CONSTANT FOR ROCK	$\mu_k = 1.19$ $\mu_o = 5°$
SIGULT σ_{ult}, ULTIMATE (UNCONFINED) COMPRESSIVE STRENGTH FOR ROCK	2900 lb/in^2
HSTEEL h_s, THICKNESS OF THE STEEL LINER	.746 in
RSUBA r_a, INSIDE RADIUS OF THE STEEL LINER	24 in
RCNMAX $r_{con_{max}}$, OUTSIDE RADIUS OF THE CONCRETE	29.75 in

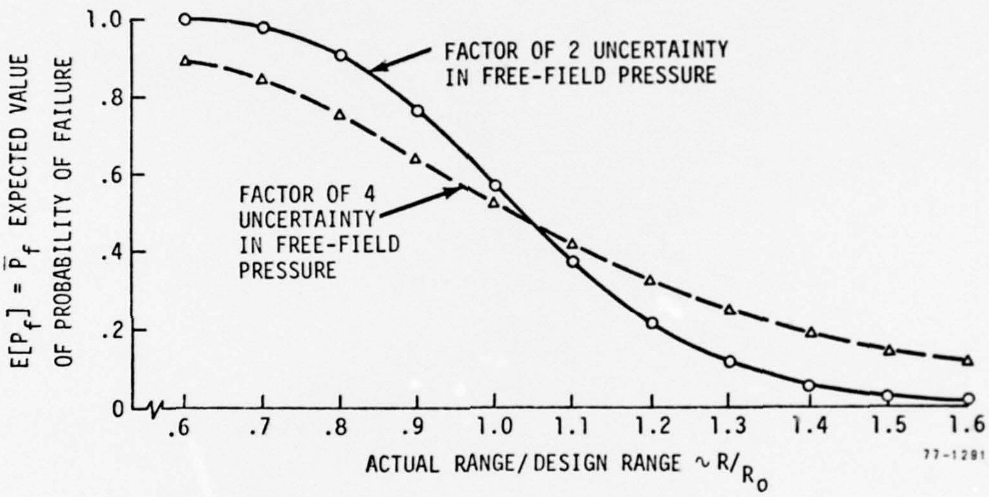

Figure 2-2. P_f - Maps Using Two Different Uncertainty Factors (From Reference 1)

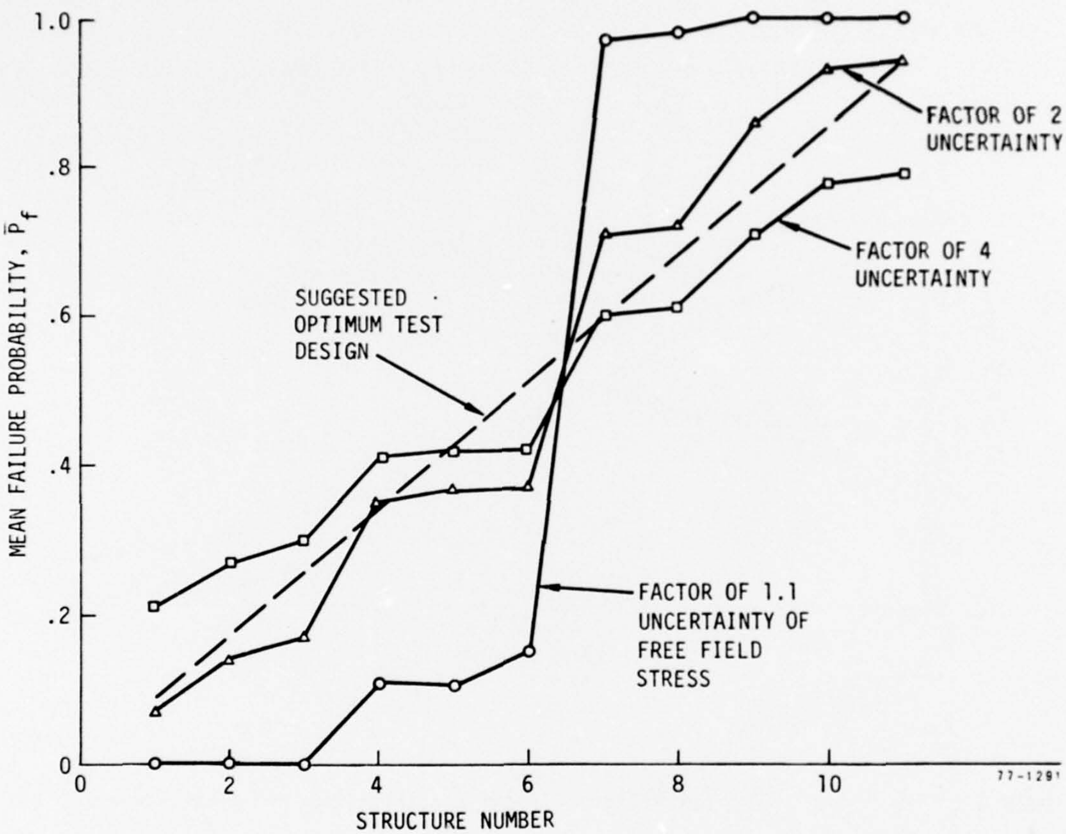

Figure 2-3. Comparison of Actual Test Design (Diablo Hawk) with Suggested Optimum Test Design

Table 2-2 describes 11 individual "events" (in the terminology of probability theory) each with a corresponding failure probability. (Note that at this point we are treating each test cylinder as independent - i.e., with no correlation among the structures or the loading. The influence of correlation is discussed in Section 4.) Viewed as 11 individual events, where the $i^{\underline{th}}$ structure has a probability of failure P_i, Collins (Reference 2) has shown how to compute the probability of exactly zero failures, etc. The mathematical expressions are presented in References 1 and 2, and calcualtions are most amenable to a simulation technique. The results of this simulation are shown in Figure 2-4 for the three uncertainty factors, namely 4, 2, and 1.1.

Note that Figure 2-4(c) for a factor of 4 uncertainty shows a symmetrical histogram, (indicating a well-balanced design) whereas Figure 2-4(a) shows that (within 90 per cent confidence limits)† 5 or 6 cylindrical structures will fail in the DIABLO HAWK test.* In fact, the probability that exactly 5 cylinders fail is approximately .65, which reflects the fact that 5 structures have individual failure probabilities $\bar{P}_f \geq .97$. These results are presented in Table 2-3 as "Predictions for DIABLO HAWK".

As indicated by the footnote in Table 2-3, thus far the effects of initial ovaling (i.e., initial imperfections) caused by the MIGHTY EPIC test have not been included, nor have the effects of correlation among structures or loading been examined. These subjects are discussed in the sections which follow.

2.3 EFFECTS OF INITIAL OVALING FROM MIGHTY EPIC TESTS

To understand the effect of initial ovaling (initial imperfections) on the behavior of lined tunnels in rock, it is instructive to consider an analogous problem, namely buckling of an imperfect column. The problem of a slightly crooked column is discussed in Reference 3, p. 13, and also in Timoshenko's well-known text (Reference 4).

For a perfect simply-supported column, of length L and having a uniform bending stiffness EI, the elastic buckling load is given by

---

†The term "confidence limits" is discussed in most texts on probability and statistics (e.g. Reference 1).

*This statement assumes that an uncertainty factor of 1.1 is representative of the Nevada Test Site.

**These results are adapted from Brush and Almroth, Reference 3.

Table 2-2. Individual Failure Probabilities for Three Uncertainty Factors

STRUCTURE NUMBER	DIMENSIONLESS RANGE, (R/R_o)	FAILURE PROBABILITY, P_f FOR 4, 2, AND 1.1 UNCERTAINTY FACTORS FOR FREEFIELD STRESS		
		$\overline{P}_{f\ 4}$	$\overline{P}_{f\ 2}$	$\overline{P}_{f\ 1.1}$
CX 10	1.24	.30	.17	.00
CX 9	1.27	.27	.14	.00
CX 8	1.36	.21	.07	.00
CX 7	1.10	.42	.37	.11
CY 23	1.10	.42	.37	.15
CY 22	1.11	.41	.35	.11
CY 16	.925	.61	.72	.97
CY 15	.930	.60	.71	.98
CY 14	.837	.71	.86	.9995
CZ 3	.775	.78	.93	.9995
CZ 1	.760	.79	.94	.9995

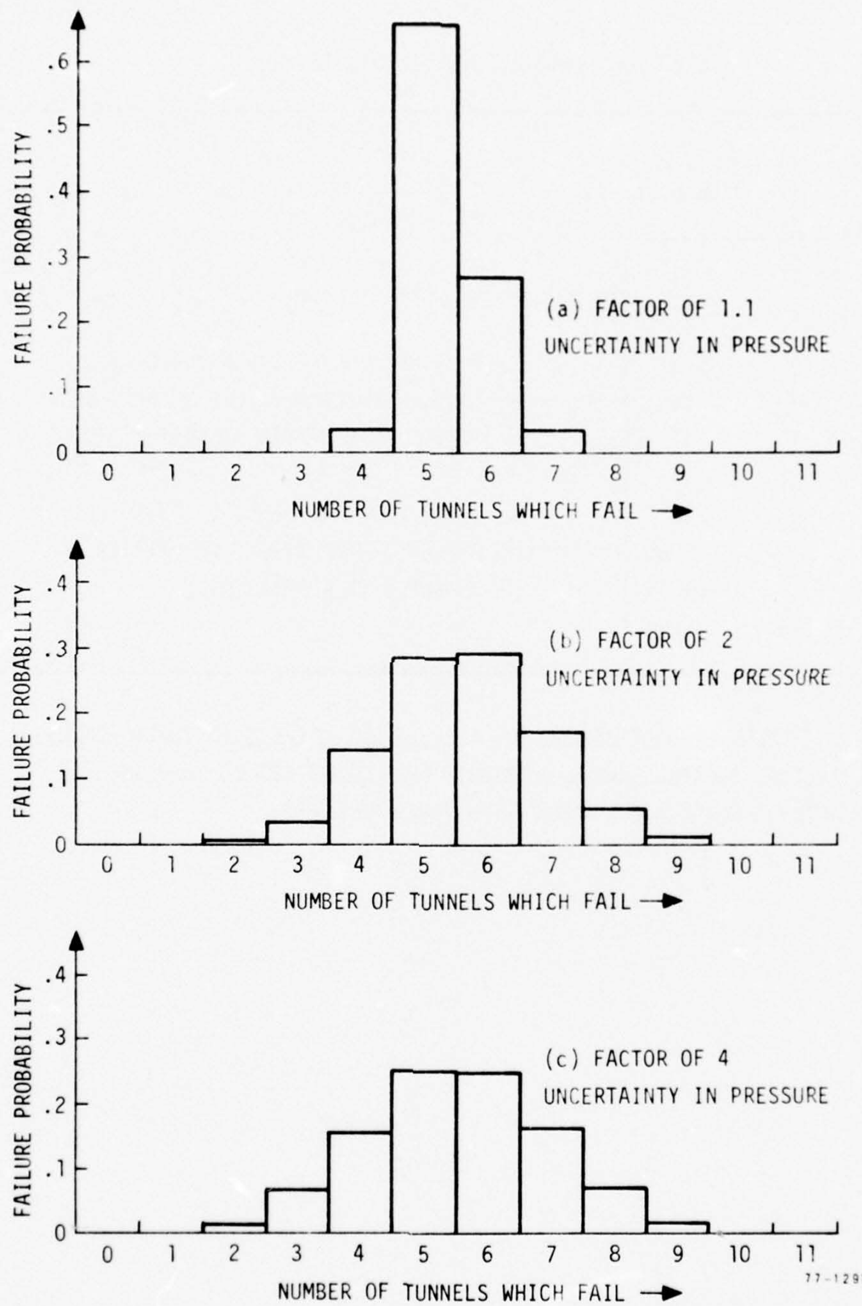

Figure 2-4. Histograms for Diablo Hawk Test (uncorrelated)

Table 2-3. Predictions for Diablo Hawk*

STRUCTURE NUMBER	FAILURE PROBABILITY $\bar{P}_{f\,1.1}$	
CX 10	.00	
CX 9	.00	NOT EXPECTED TO FAIL
CX 8	.00	
CX 7	.11	
CY 23	.15	POSSIBLE ONE OF THESE THREE WILL FAIL, WITH DAMAGE LIKE SLIGHT FLATTENING EXPERIENCED IN MIGHTY EPIC.
CY 22	.11	
CY 16	.97	
CY 15	.98	
CY 14	.9995	THESE FIVE ARE EXPECTED TO FAIL. EXPECT SEVERE DAMAGE COMPARABLE TO FAILURES IN MIGHTY EPIC.
CZ 3	.9995	
CZ 1	.9995	

*NOTE THAT THESE PREDICTIONS DO NOT INCLUDE THE EFFECTS OF IMPERFECTIONS, (E.G., INITIAL OVALING CAUSED BY MIGHTY EPIC TEST) NOR DO THEY INCLUDE THE EFFECTS OF CORRELATION AMONG STRUCTURES OR LOADS

For a perfect simply-supported column, of length L and having a uniform bending stiffness EI, the elastic buckling load is given by

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{2-1}$$

When the column is <u>initially imperfect</u>,** say it has an initial deflection

$$w^* = C_1^* \sin \frac{\pi x}{L} \tag{2-2}$$

then the deflection in the buckling mode is given by

$$w(x) = \frac{PC_1^* \sin \frac{\pi x}{L}}{\frac{\pi^2 EI}{L^2} - P} \tag{2-3}$$

where C_1* is the amplitude of the imperfection. Note that equation (2-3) predicts that the lateral deflection, w, at the middle of the column (i.e., x = L/2) becomes unbounded as the load P approaches $\pi^2 EI/L^2$. Qualitatively, the effect of initial imperfections is shown in Figure 2-5.

When C_1*, the initial imperfection, is small, the behavior approaches that of a perfect column. As C_1* becomes larger, the lateral deflection grows more rapidly, <u>but in all cases</u>, the lateral deflection increases without bound as P approaches $P_{cr} = \pi^2 EI/L^2$, the buckling load of the perfect column. This result, namely that the maximum load which the column will carry is not strongly-dependent on the initial imperfection (C_1*) has led researchers to classify columns as "insensitive" to initial imperfections.

As might be imagined, not all structures are insensitive to initial imperfections. Figure 2-6 qualitatively shows the effect of imperfections on the buckling of an axially compressed cylindrical shell (Reference 3). Certain shell structures are notorious for their sensitivity to initial imperfections, and this fact usually is manifested by a significant discrepancy between (perfect) theory and (imperfect) experiments. With respect to the problem at hand, namely failure of lined tunnels in rock, the next step is to determine whether the tunnels are relatively insensitive (like the column) or highly-sensitive (like the axially-loaded shell). The results available (Reference 3) indicate that tunnels loaded by external (hydrostatic) pressure are relatively insensitive to initial imperfections, at least for the (MIGHTY EPIC/DIABLO HAWK) geometries.

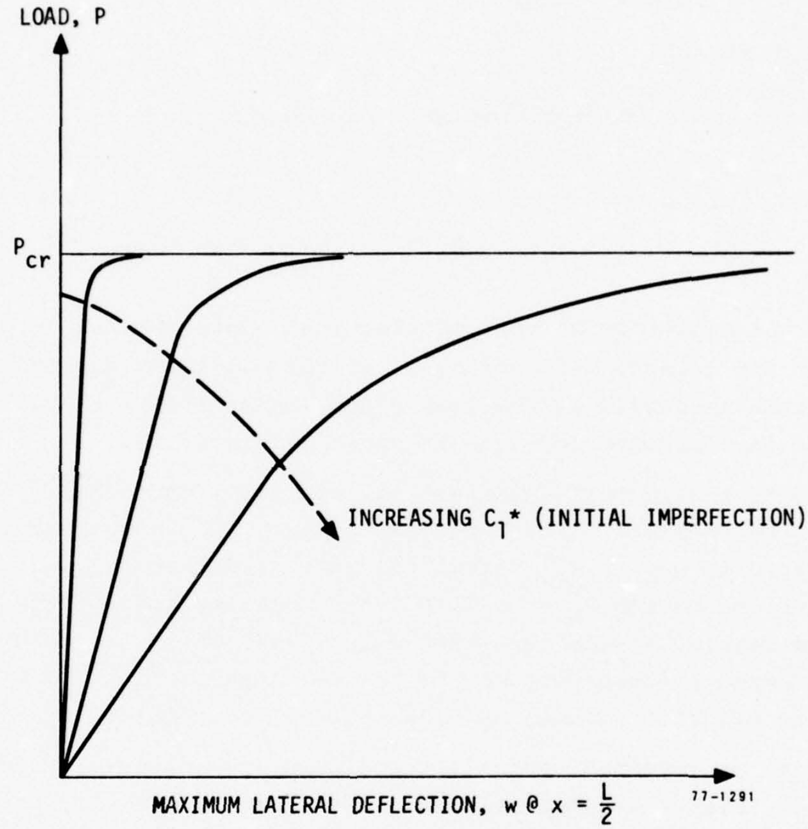

Figure 2-5. Load-Deflection Curves for an Imperfect Column

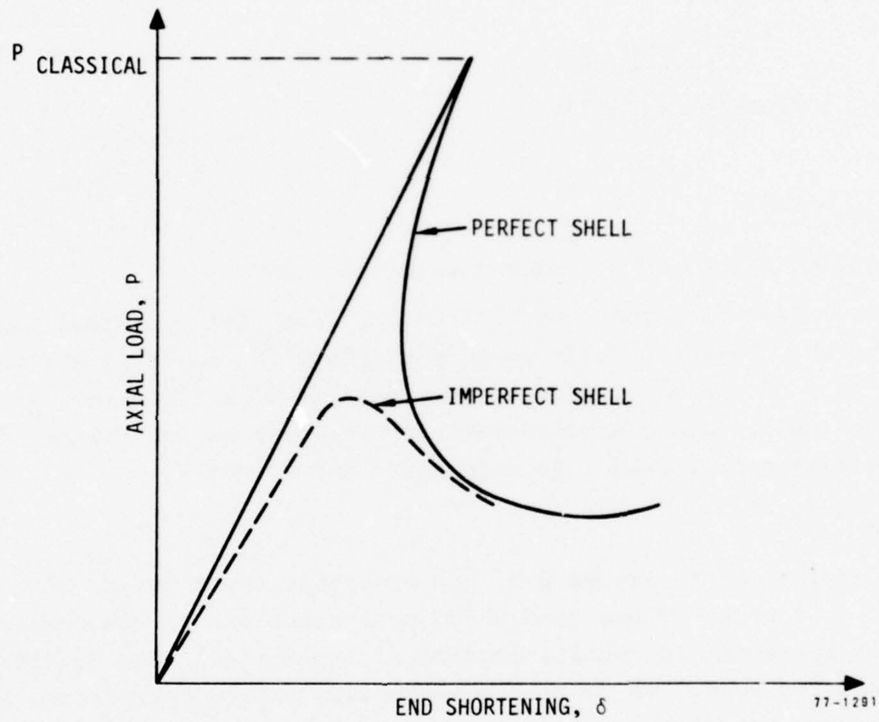

Figure 2-6. Load-Deflection Curves for an Axially-Compressed Cylindrical Shell

For example, Figure 2-7 shows the comparison between theory (the solid line) and experiment (the open dots) for buckling pressure $\overline{p}$ vs. the Batdorf parameter, Z. When Z is computed for the test cylinders, one has

$$Z = \frac{L^2}{ah} (1 - \nu^2)^{1/2} \tag{2-4}$$

where

L = 16 ft (length)
a = 2 ft (radius)
h = .75 in (thickness)
ν = .3 (Poisson's ratio)

which gives

$$Z_{.75} = 1953$$

when a steel thickness of .75 inches is used.

This calculation does not reflect the fact that the steel liner is attached to the concrete, which is much thicker. By equating the bending stiffness (EI) of the steel-and-concrete to that of a completely steel "equivalent" shell, one can conservatively estimate an equivalent thickness, h_{eq}, as less than 12 inches. In this case, one computes

$$Z_{12} = 122 \tag{2-5b}$$

Referring now to Figure 2-7, one sees that the Z values of equations (2-5) fall in a region where theory and experiment are in reasonable agreement. This agreement is usually lacking if the structure is sensitive to initial imperfections. To further substantiate this point, Figure 2-8 shows a theoretical "imperfection sensitivity parameter", a_2, plotted vs. Z for cylindrical shells loaded by external pressure. Note that for $Z > 100$, the sensitivity parameter a_2 is small, (tending toward zero) which indicates a lack of sensitivity to initial imperfections.

The net result of this discussion is that the (MIGHTY EPIC/DIABLO HAWK) test cylinders are not expected to be sensitive to initial imperfections. Thus, although MIGHTY EPIC produced some (relatively slight) initial ovaling in the (undamaged) test cylinders, these imperfections are not epxected to significantly change the DIABLO HAWK predictions. It is known that the effect of the imperfections will be to increase the expected probability of failure, $\overline{P}_f$, but such changes are <u>not</u> expected

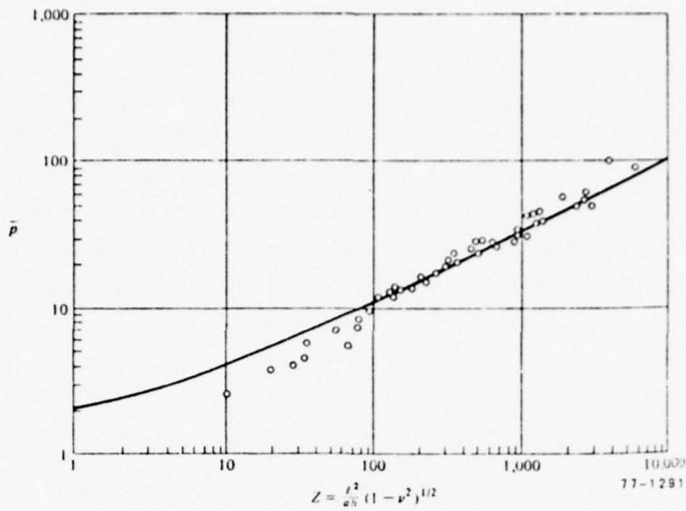

Figure 2-7. Comparison of Theoretical and Experimental Values for Cylinders Subjected to Hydrostatic Pressure

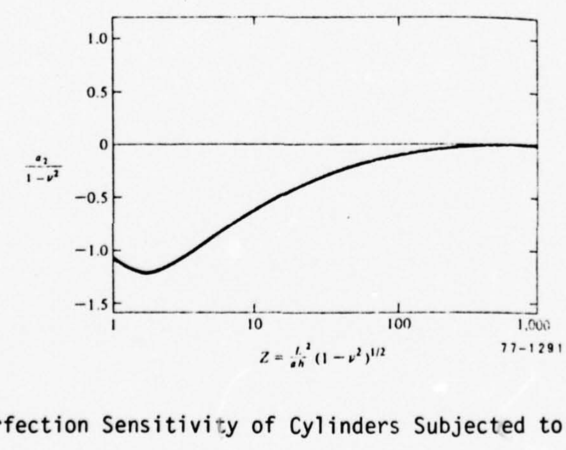

Figure 2-8. Imperfection Sensitivity of Cylinders Subjected to Lateral Pressure

to be significant. Again the reader is cautioned that the effect of pre-stress in the rock medium has not been inlcluded in this discussion, and this pre-stress effect may be important.

Referring back to Figure 2-4 for a moment, one sees that 5 or 6 cylindrical structures are expected to fail in the DIABLO HAWK test, and Table 2-3 indicates that 5 cylinders will be "severely" damaged. Armed with this knowledge, one might pose the question, "How can we adjust the DIABLO HAWK test structures to improve the probability of obtaining successful test data?" This question is addressed in Section 3. of this report.

3. RECOMMENDATIONS FOR THE DIABLO HAWK TEST

3.1 RECOMMENDATIONS

The test planners for the (MIGHTY EPIC/DIABLO HAWK) structures test had certain design goals in mind when they selected the sizes and placement of their structures. Although all of their design goals are not known to the writer, two goals which were desired are

- The desire to load the same structures more than once, i.e., multiple loadings, and
- The desire to propagate a second shock pulse through the tuff/grout medium, which may have been fractured and/or compressed by the first test.

These two test goals have an impact on the realistic (i.e., practical) changes which can be considered to improve the probability of acquiring useful data in the DIABLO HAWK test. Section 2 tells the story of what is expected on DIABLO HAWK:

- Five cylinders are expected to fail severely, similar to those most extensively damaged in MIGHTY EPIC, and
- One other cylinder (out of three candidates) may experience slight buckling.

Faced with these goals and predictions, the following recommendation is made:

- Add one or more cylindrical structures with a .5 probability of failure.

The reasons behind this recommendation are given in the paragraphs which follow, along with other alternatives that appear less practical. Section 3.2 also contains an example decision analysis which also indicates the need for one more structure.

Referring to either Figure 2-3 or Table 2-2, for a factor of 1.1 uncertainty, one sees a less than optimally designed experiment for the cylindrical tunnel structures. Five structures have $\overline{P}_f \geq .97$; three structures have $\overline{P}_f$ .00, and the other structures have mean failure probabilities of .11, .11, and .15 respectively. To improve the test design, it is recommended that DNA consider adding at least one more tunnel, designed with an intermediate failure probability.

Such a test matrix will give a more optimum design and is expected to increase the probability of obtaining useful data. If no changes are made in the DIABLO HAWK test, it is very probable that five (5) structures will be severely damaged, one just slightly damaged, and the other five will be basically unaffected by the event. The proposed additional structure is expected to produce an intermediate damage which will increase the information and useful data obtained.

Other alternatives which might be considered but which have inherent difficulties are listed in Table 3-1, along with a qualitative estimate of their attendant cost. It is evident that further study of the problem is required before an intelligent final choice can be made among the various alternatives. In that regard, it may be premature to suggest adding more tunnels to improve the test, but the additional tunnels appear to be the leading candidate for improvement as of this writing.

The reader should note that the predictions made in Section 2 and re-stated herein do not include the effects of "pre-shocking" on the tuff. The MIGHTY EPIC test fractured some of the rock, compressed it, etc,. and the predictions of this report have not included these effects. It is assumed that these effects are negligible or of minor significance; however, this assumption may prove to be incorrect.

3.2 EXAMPLE DECISION ANALYSIS FOR SELECTING AN OPTIMUM NUMBER OF ADDITIONAL STRUCTURES

Although the DIABLO HAWK test is already set, and further changes will be difficult, it is of interest to determine whether additional cylinders would be of value in the future application of data from the test. The problem with the current configuration for DIABLO HAWK is that the cylinders are either failing with a high probability or with a very low probability, and there is very limited data as to the average probability, i.e., the level of structure strength where the probability of failure would be around .5. Consider the problem shown in Figure 3-1. There is a design level μ_x in a future system which resulted from the conclusions about cylinder strength in the MIGHTY EPIC and the DIABLO HAWK tests. There is an uncertainty about this mean strength, σ_0, resulting from the lack of information from failures in the tests relating to the $P_f = .5$ range. Consequently, a normal distribution is shown, representing the uncertainty in the true capability of the structure. Since this uncertainty exists at the time of the design of the future system, the designer must take a conservative position and possibly

Table 3-1. Alternatives Considered for Improving the DIABLO HAWK Test

ALTERNATIVE	EXPECTED RESULTS	UNDESIRABLE SIDE-EFFECTS	PRACTICALITY
1. Put stiffening rings in the cylinders with high failure probabilities	Will lower $\overline{P}_f$, improving the test	Changes the structure. Runs counter to the multiple-loading objective	Moderately expensive
2. Re-configure the test, moving existing cylinders to new ranges	Will lower the high $\overline{P}_f$'s, raise the low $\overline{P}_f$'s, result in near optimum test	Disturbs the rock/grout surrounding the structures. May alter the preshocked effects of fractured rock, etc.	Expected to be very expensive
3. Fill with water and seal off the cylinders with high failure probabilities	Will lower $\overline{P}_f$, improving the test	Changes the structure. Same objection as Item 1 above	Perhaps the least costly alternative considered here
4. Add a few new cylinders, with linearly varying $\overline{P}_f$	Will improve the test	Tunnels will be required in preshocked (i.e., *non-virgin*) rock. (May not be a problem, however)	Moderately expensive

overdesign, because he is not sure of the true mean capability of the cylinders. The resulting cost to the future system is in additional materials which would be needed to give the additional strength. If, indeed, the cylinder were stronger than estimated by the designer because of his lack of uncertainty, then there would be an economic opportunity loss (a spending of money which did not need to be spent), due to the uncertainty.

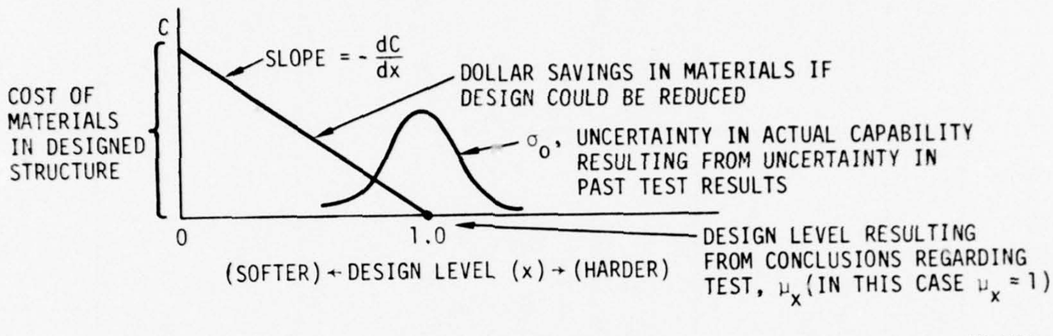

Figure 3-1. Economic Opportunity Loss as a Function of Mean Design Level and Uncertainty

The average financial loss due to the uncertainty can be expressed as an "expected opportunity loss" as shown in Equation (3-1)

$$E[\text{Opportunity Loss}] = EOL = \int_o^{\mu_x} \left(C - \frac{C}{\mu_x} x\right) \frac{1}{\sqrt{2\pi}\, \sigma_o} e^{-\left(\frac{(x-\mu_x)^2}{2\, \sigma_o^{\,2}}\right)} dx \qquad (3\text{-}1)$$

Note there is no opportunity loss if the design level should have actually been higher. On the other hand this would increase P_f which is also undesirable. Hence another line should start at $x = \mu_x$ and have a positive slope based on some financial interpretation of increased probability of failure. At this point we will suggest, for purposes of demonstration, that this second line be ignored and that the problem be restricted to that shown in Figure 3-1.

The uncertainty in the proper design level can be reduced by adding structures to the next test which are designed such that they have a probability of failure equal to approximately .5. It would be best to have a range of failure probabilities from .2 or .3 to .7 or .8, but regardless the results should yield a mean design level, $\bar{x}$, which would represent the best estimate of the average design level.

Assume that the uncertainty in the failure level of any specific structure (σ_s) can be obtained from the fragility curve. Then the standard error of the estimate of the mean is $\sigma_x = \sigma_s/\sqrt{n}$, where n is the number of structures being tested.

To determine the optimum number of tests required to minimize cost (combined system cost and testing cost), assume that the normal distribution in Figure 3-1 represents our "prior" knowledge of the uncertainty and theoretically we could afford to pay up to the number of dollars in the EOL for additional tests _if_ the results from the additional tests were able to reduce to zero our uncertainty in the estimate of the true mean. Unfortunately, the results of the tests will still produce uncertainty in the estimate of the mean, although less than the original.

If it is assumed that all the distributions involved are normal (i.e., Gaussian), then it can be shown that the revised estimate of the uncertainty in the estimate of the true mean can be written as follows (Ref. 14, 15)

$$\frac{1}{\sigma_r^2} = \frac{1}{\sigma_o^2} + \frac{1}{\sigma_x^2} \tag{3-2}$$

where σ_r is the revised uncertainty in the estimate of the true mean

σ_o is the prior uncertainty

$\sigma_{\bar{x}}$ is the uncertainty in $\bar{x}$ resulting from sampling (additional tests)

We are interested in the improvement (reduction) in the uncertainty (σ_I) which can be written as follows:

$$\sigma_I = \sqrt{\sigma_o^2 - \sigma_r^2}$$

$$= \sqrt{\sigma_o^2 - \frac{\sigma_x^2\,\sigma_o^2}{\sigma_o^2 + \sigma_x^2}}$$

$$= \sigma_o\sqrt{\frac{\sigma_o^2}{\sigma_o^2 + \sigma_x^2}} \tag{3-3}$$

σ_I, as shown, can now be substituted into Equation (3-1) to provide the economic value of the sample information. Since $\sigma_{\bar{x}} = \sigma_s/\sqrt{n}$, this value will change with the number of tests. The economic benefit from the tests will be the economic value of the sample (test) information minus the cost of the tests.

For example, assume the following

μ_x = 1 (reference level for the design)

σ_o = .056 (estimated by evaluating the uncertainty range in the mean estimate after reviewing the results of MIGHTY EPIC)

$\frac{dC}{dx} = \frac{5,000,000}{1}$ (\$5 million material cost/100% change in strength)

σ_c = .10 (typical from fragility curves in Reference 1).

From Equation (1), EOL = \$111,704. To find the expected average gain from the test information, use the following formulation.

E[Value of Test Information] = EVTI

$$= \frac{dC}{dx} \frac{1}{\sqrt{2\Pi}\,\sigma_I} \int_{-\infty}^{\mu_x} (x-\mu_x)\, e^{-\frac{(x-\mu_x)^2}{2\sigma_I^2}}\, dx$$

$$= \frac{dC}{dx}\, \sigma_I \frac{1}{\sqrt{2\Pi}}$$

$$= \frac{dC}{dx} \frac{\sigma_o}{\sqrt{2\Pi}} \sqrt{\frac{\sigma_o^2}{\sigma_o^2 + \sigma_c^2/n}} \qquad (3\text{-}4)$$

If the cost per additional test (cylinder) is C_T = \$50,000 then the net gain from additional testing is

$$\text{Net gain from additional tests} = G = \text{EVTI}(n) - n\,C_T = \frac{dC}{dx} \frac{\sigma_o}{\sqrt{2\Pi}} \sqrt{\frac{\sigma_o^2}{\sigma_o^2 + \frac{\sigma_c^2}{n}}} - n\,C_T \qquad (3\text{-}5)$$

On substituting the assumed values of μ_x, σ_o, etc. into equations (3-4) and (3-5), it is possible to determine the net gain financially from the addition of structures to the test. The results for this example are shown in Table 3-2. It can be seen that one additional structure would give a small net gain over no structures or over two structures which would cost too much compared to the benefit. By changing the material costs of the future facilities, it is possible to do an analysis of the optimum number of additional structures as a function of the cost of material in the future structures. This is shown in Figure 3-2. Figure 3-3 shows the average net financial gain due to using optimum number of additional tests as a function of the costs of materials in future systems. Obviously, there is a considerable gain as the future systems become more and more expensive.

From this brief analysis, it appears that, at the very most, one additional structure with a predicted failure probability of .5 would be desirable in the DIABLO HAWK tests. However, it appears that the net benefit from the additional structure will be small, and would not merit the delaying of the start of the tests, which would add even further costs.

n	EVTI(n)	n C_T	EVTI(n)-n C_T
0	0	0	0
1	54,579	50,000	4,579
2	69,350	100,000	-30,650

Table 3-2. Net Gain from Additional Structures

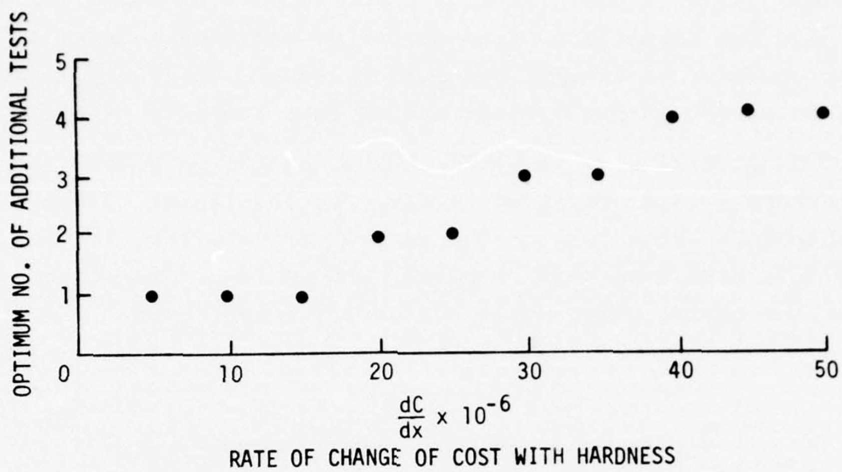

Figure 3-2. Optimum Number of Tests as a Function of Future Facility Cost

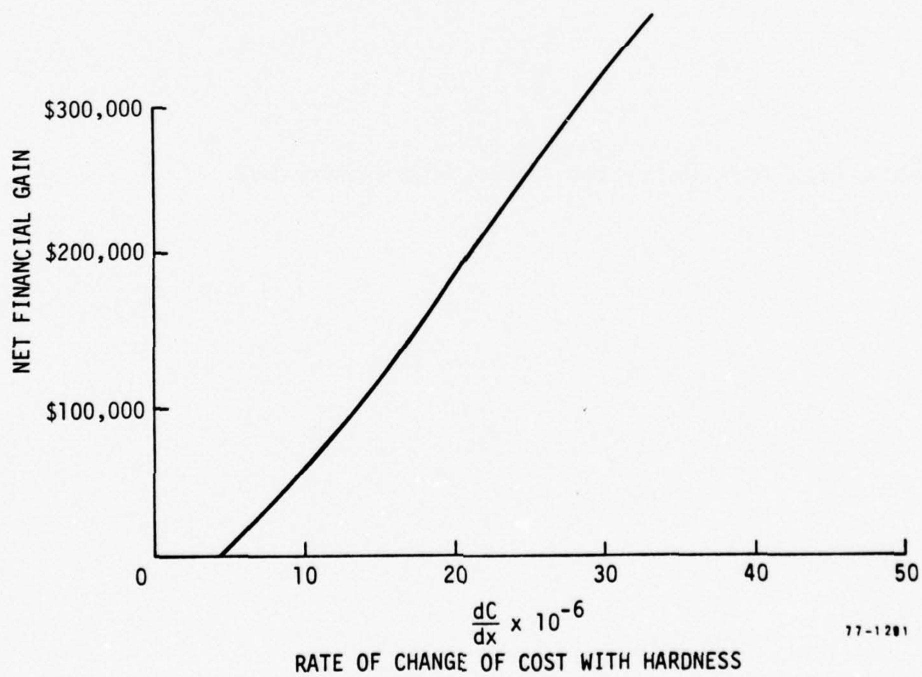

Figure 3-3. Average Net Financial Gain Due to Using Optimum Number of Additional Tests

4. EFFECTS OF CORRELATION

4.1 BACKGROUND

As indicated previously (Reference 1) correlation is a significant but frequently omitted aspect of a statistical evaluation. There are two prime sources of correlation in the lined tunnel problem:

- The free-field stress level, if high, will be high everywhere and, if low, will be low everywhere. There may be some slight variations in the value from point to point (at the same range), but the heavy trend will exist.
- Strength of materials such as the reinforced concrete will also tend to be heavily correlated from structure to structure because of the commonality of material source and manufacturing and aging processes.

The correlation of strength of structures and the correlation of the free-field stress have no relationship to each other. Thus, in a simulation procedure, one random variable is generated for the pressure (and appropriately scaled for all the tunnels), and another (independent) random variable is generated for the material strength and likewise used for all structures.

The easiest problem to consider is the case where the structures are fully-correlated for all the random (structural property) variables. That is, if the concrete is especially strong, it will be strong for all the test tunnels. Similarly, if the steel is slightly too thick, it will be too thick for all the structures, and so on for E_{rock}, ν_{rock}, σ_{ult}, etc. This assumption of full correlation (in all the random variables) will clearly give an upper bound on the effects of correlation. The actual physical problem clearly lies somewhere between fully correlated (considered herein) and totally uncorrelated (considered previously in Reference 1).

The generation of failure probabilities for the fully-correlated problem involved developing a small FØRTRAN program, which is outlined in Figure 4-1. Beginning on the left of Figure 4-1, one has two independent random number generators. Since P_f (on each fragility curve) lies between 0 and 1, the procedure is to generate a random number between [0,1]. The random number has a uniform probability distribution on the [0,1] interval. Using the random number (called "RANDY" in the FØRTRAN program) one enters the fragility curve* and reads off a structural capability, call it (σ/σ_o).

*Actually, there are four slightly different fragility curves in the program, corresponding to different concrete thicknesses. See Figures 4-5 through 4-8 of Reference 1 .

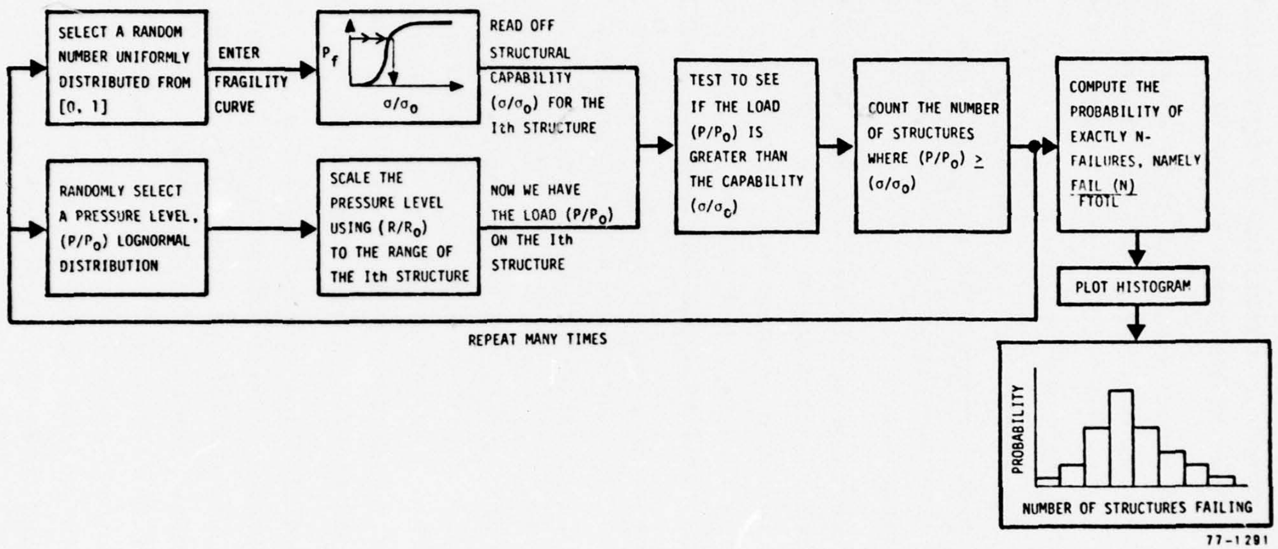

Figure 4-1. Flow Chart of the Fully-Correlated Computer Analysis

Similarly, one generates a log-normal pressure distribution, corresponding to a dimensionless range $R/R_o = 1.0$. Then, knowing the range of the structure, one proceeds to scale the pressure variable to give (P/P_o) at the $I^{\underline{th}}$ structure. Not shown in the Figure is a DO-LOOP (over I, where I ranges from 1 to the total number of structures, NSTRUC). Within the DO-LOOP a test is made to see if

$$\left(P/P_o\right)_I > \left(\sigma/\sigma_o\right)_I \tag{4-1}$$

If equation (4-1) is true, then the $I^{\underline{th}}$ structure will fail, and a counter is incremented. For example, in a single pass through the Monte Carlo Loop, 5 structures might "fail" and 9 survive. In this case, the variable FAIL (5) is incremented by one.

The result of this procedure (when repeated many times) is to populate a frequency distribution such as shown in Figure 4-2. Note that on each pass through the Monte Carlo Loop, the random number generated for the structure (P_f) is used for <u>all</u> the tunnels (hence the tunnels are 100 per cent correlated) and the random pressure is scaled (using $P \sim R^{-2}$) for <u>all</u> ranges (hence the pressure loading is 100 per cent correlated).

The computer program was relatively straightforward, and it is thought to be error-free. As a check problem, one range [$(R/R_o)_1$ say] was given its normal value, and all the other ranges were set to 1000. This check-out procedure corresponds to a single structure within the range of the weapon, and all the other structures out-of-range. The result is to produce the (uncorrelated) failure probability of a single (independent) tunnel, at range $(R/R_o)_1$. This value for $\overline{P}_f$ was then checked against the curve in Figure 2-2, and agreement was obtained.

After thus verifying that the program was working correctly, several runs were made, using three different "factors of uncertainty" in the log-normal pressure distribution. The primary result of correlating the structures and correlating the loads is that the probability distributions (histograms) begin to develop significant "tails" and the standard deviation increases. This result is in agreement with physical intuition about the problem, since "If the structures are all extra-strong, they'll all be more likely to survive, and if they are all extra-weak, they'll all be more likely to fail." Similar arguments for emphasing the extremes of the probability distributions apply when the correlation of the pressure is considered.

Individual results for MIGHTY EPIC and DIABLO HAWK are discussed in the sections which follow.

4.2 EFFECT OF CORRELATION ON DIABLO HAWK PREDICTIONS

The results presented in Section 2.2 (cf. Figure 2-4a through 2-4c) are based upon un-correlated random variables. Thus the concrete strength of one cylinder might be high when the concrete strength of its neighbor is low, etc. Similarly, the results of Section 2.2 assume that the pressure loading (i.e., free-field stress) is also random from cylinder-to-cylinder.

From a practical, physical standpoint, the assumption that the cylinders are all uncorrelated (with regard to concrete strength, rock properties, etc.) is not correct. Presumably, the same construction crew made each of the cylinders, some were made on the same day, they were made using the same techniques, etc. Thus, it is anticipated that certain properties of the structures will be highly correlated. Similarly, the pressure loading (from the nuclear event) is also expected to be highly correlated, such that a high free-field stress on one structure implies a high free-field stress on its neighbor, etc.

Note that the pressure loading is uncorrelated with the material properties, however. That is, the high concrete strengths made by the construction crew are totally uncorrelated with the effects produced by the weapon. From a computational standpoint, the simplest case to consider is that where

(i) All the material properties, rock strengths, concrete strength, etc. are fully-correlated from structure-to-structure,

(ii) The pressure loading is fully-correlated from structure-to-structure, and

(iii) There is no correlation between material strengths, cylinder properties, and the applied loading.

This problem was solved computationally by using two independent random number generators via the computer program outlined in Figure 4-1. The results of these calculations are shown in Figure 4-2 through 4-4, which were each computed for a different factor of uncertainty in the free-field pressure. Referring to Figure 4-2 and comparing it with Figure 2-4a one observes that the effect of correlation has been to increase the probability of exactly 3 failures and exactly 8 failures. Previously, the probability of exactly 6 failures was quite high (see Figure 2-4a). Thus,

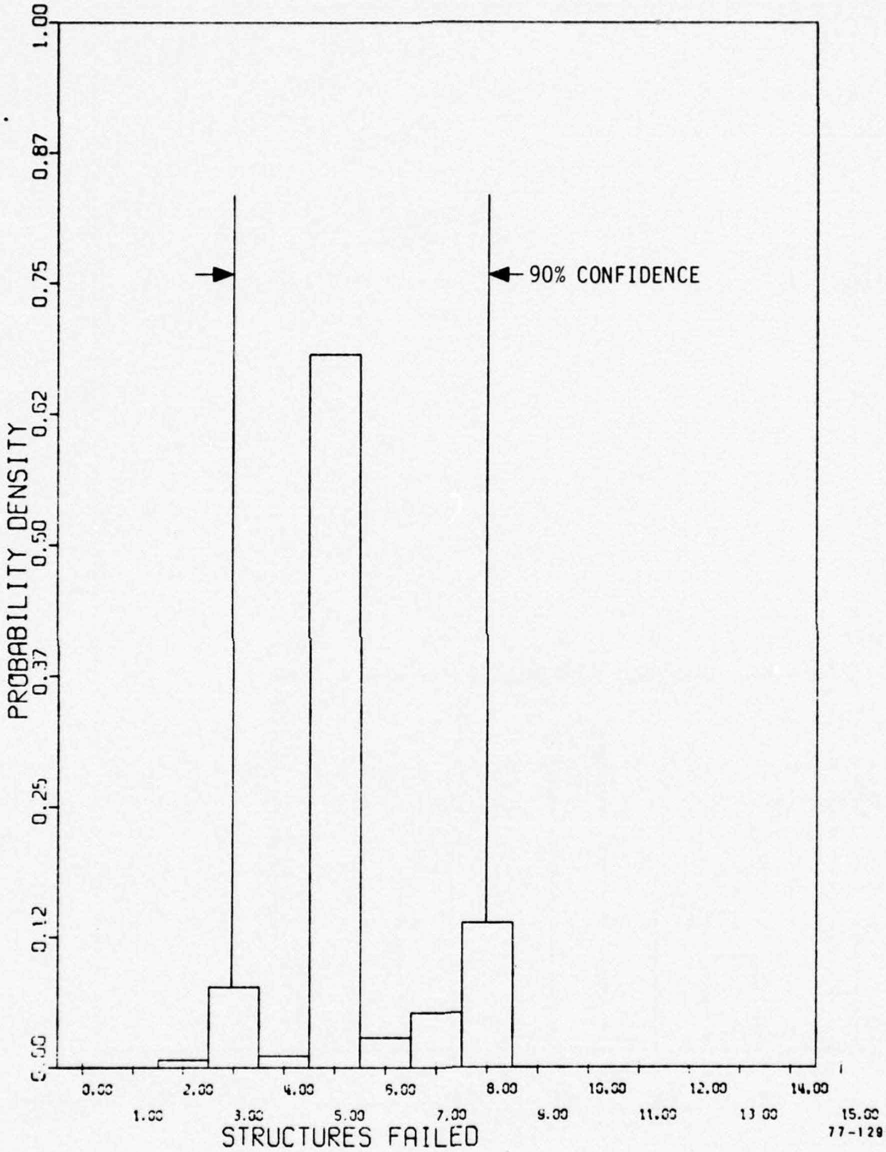

Figure 4-2. Fully-Correlated Histogram for Diablo Hawk
(Factor of 1.1 Uncertainty in Pressure)

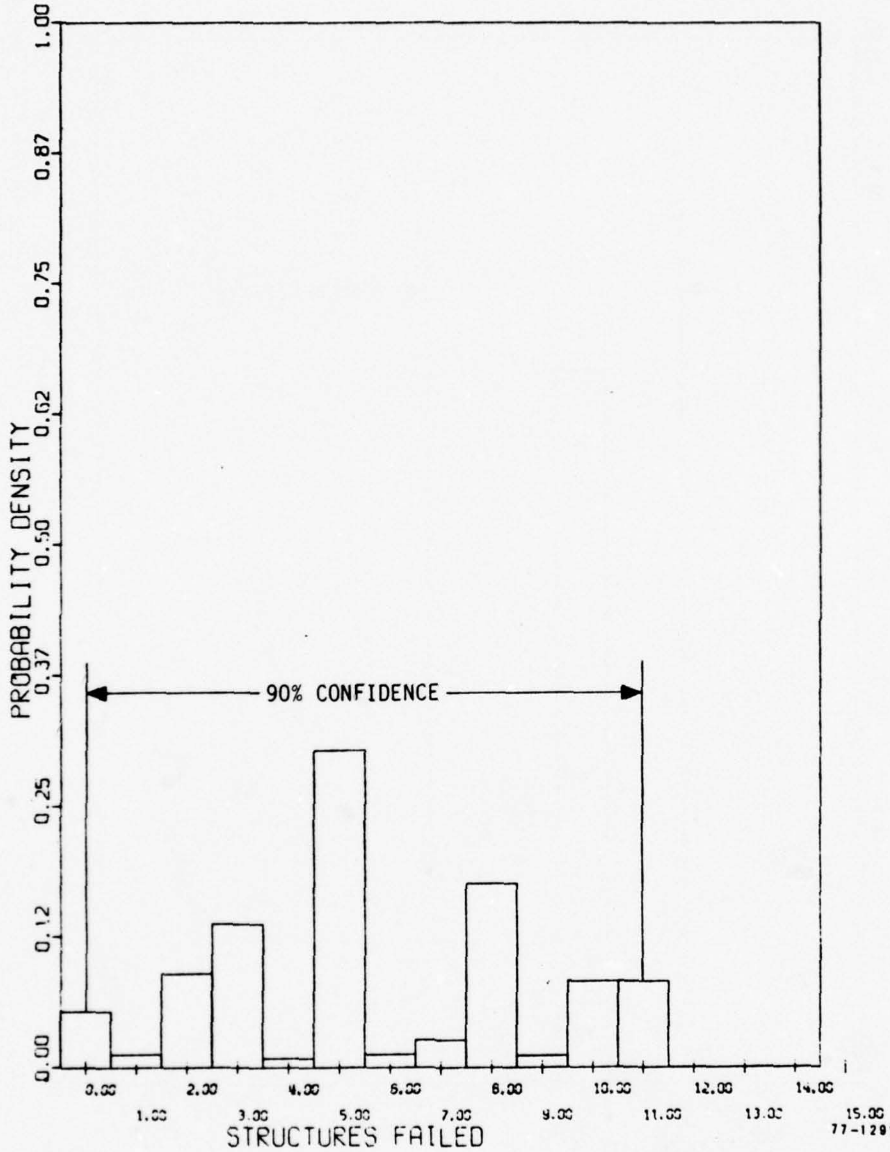

Figure 4-3. Fully-Correlated Histogram for Diablo Hawk
(Factor of 2 Uncertainty in Pressure)

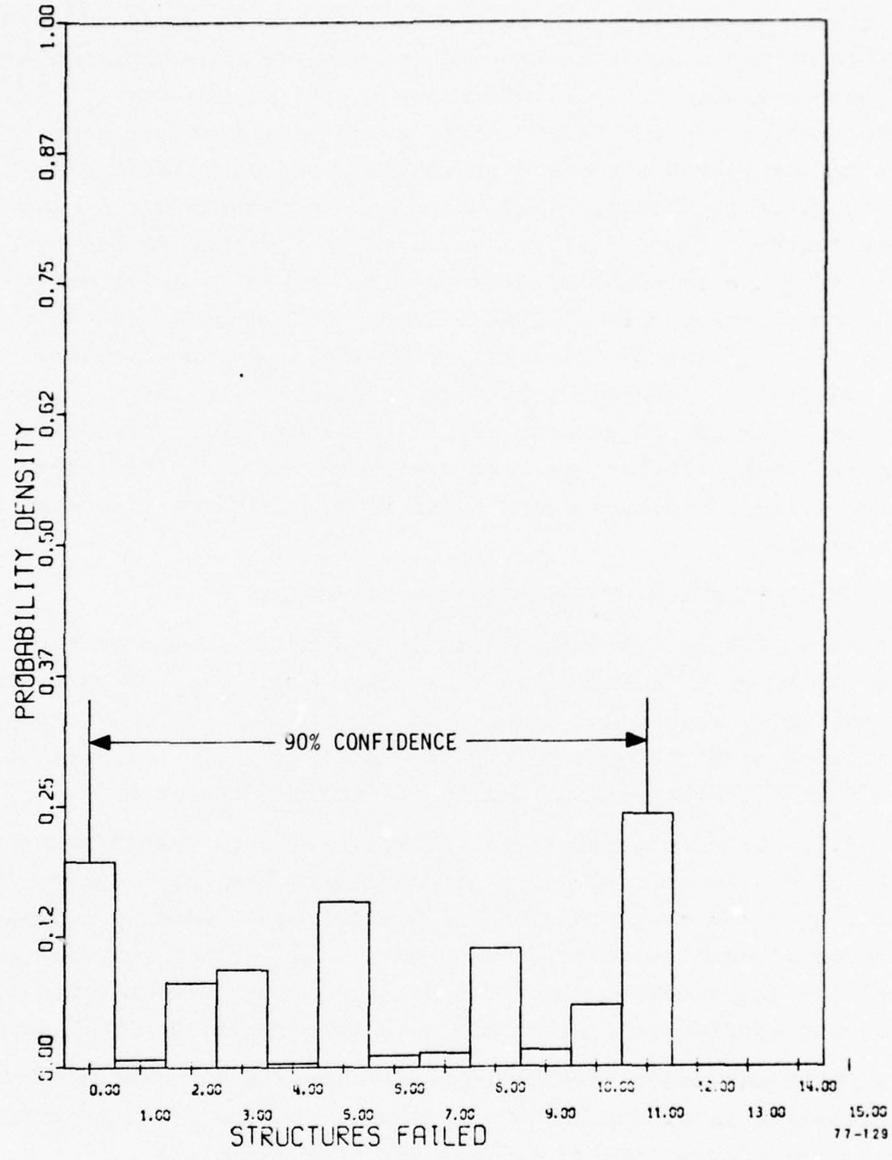

Figure 4-4. Fully-Correlated Histogram for Diablo Hawk
(Factor of 4 Uncertainty in Pressure)

the effect of correlation has been to modify the extremes of the distribution and increase the standard deviation.

Similar results are noted when Figure 2-4(b) is compared with Figure 4-3. Again the effect of correlation has been to modify the extremes of the distribution. This effect if most pronounced for the larger uncertainties in the free-field pressure (compare Figure 2-4(c) with Figure 4-4). With respect to the predictions for DIABLO HAWK, since the effect of correlation has been to widen or spread out the distribution, the range on the predicted number of failures is increased. Thus, although in Section 2.2 it was predicted that "either 5 or 6 failures would occur,* within 90 per cent confidence limits," the fully-correlated results of Figure 4-2 indicate that between 3 and 8 structures will fail in the DIABLO HAWK test, within 90 per cent confidence limits. Similarly, for the higher uncertainty factors, the range on the number of predicted failures increases. These effects of correlation and of uncertainty in the free-field pressure were also demonstrated when calculations were preformed for the MIGHTY EPIC cylinders. The fully-correlated results for MIGHTY EPIC are discussed in the following section.

4.3 EFFECT OF CORRELATION ON THE MIGHTY EPIC RESULTS

Referring to Figure 4-5 (taken from Reference 1) one sees that the probability of 4, 5, or 6 failures was relatively high, when no correlation was used. Conversely, Figure 4-6 shows that the probability of exactly 5 failures is reduced, and the probability of exactly 8 or 10 failures is increased, when the structures and loading are fully-correlated.

Previously, without correlations, it was predicted that "from 4 to 6 structures will fail, within 90 per cent confidence limits." Again, it is noted from Figure 4-6 that the range of the probable number of cylinders failing is increased when correlation is used. Thus, from Figure 4-6 one can determine that the probable number of failures lies between 4 and 10, within 90 per cent confidence limits, for a fully-correlated MIGHTY EPIC test.

With higher uncertainty factors (for the free-field pressure) the effect of correlation is to emphasize the extremes of the distributions, as noted previously. (See Figures 4-7 and 4-8, for example.)

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*For a factor of 1.1 uncertainty in pressure.

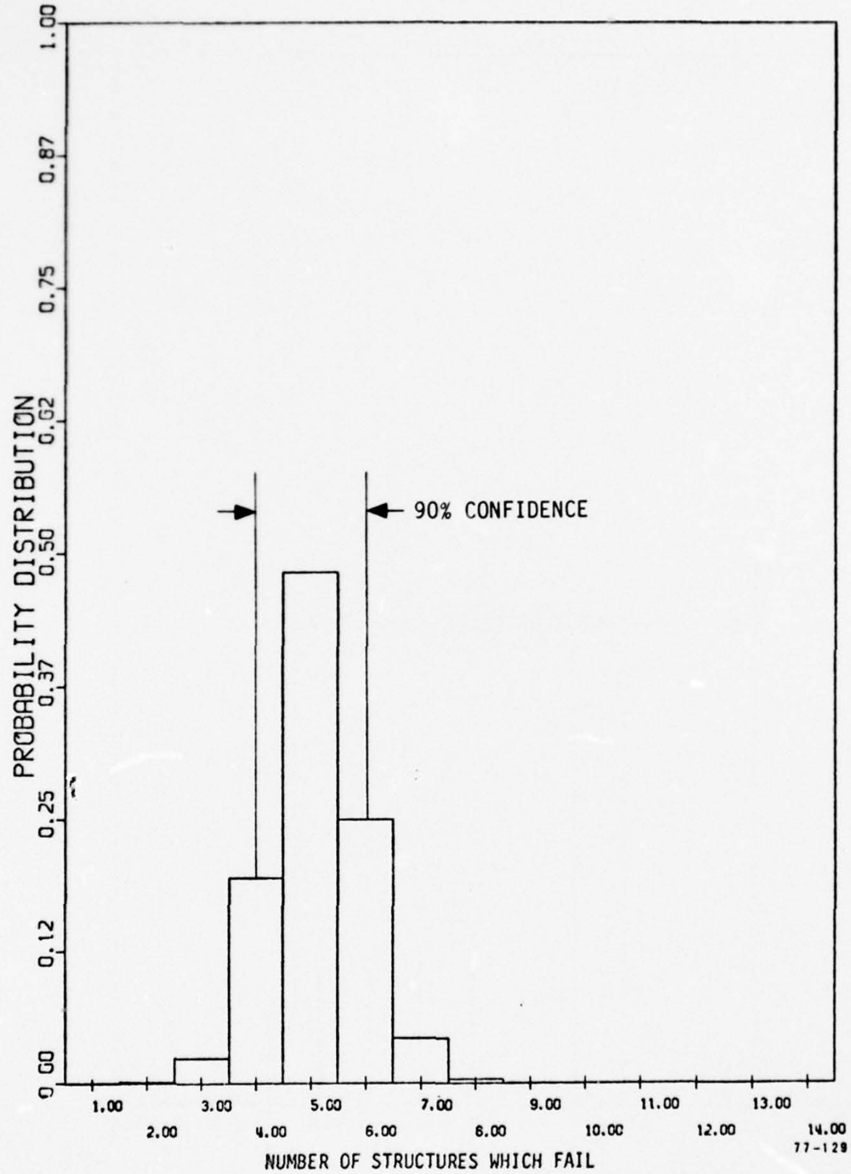

Figure 4-5. Un-correlated Histogram for Might Epic
(Factor of 1.1 Uncertainty in Pressure)
[Reference 1]

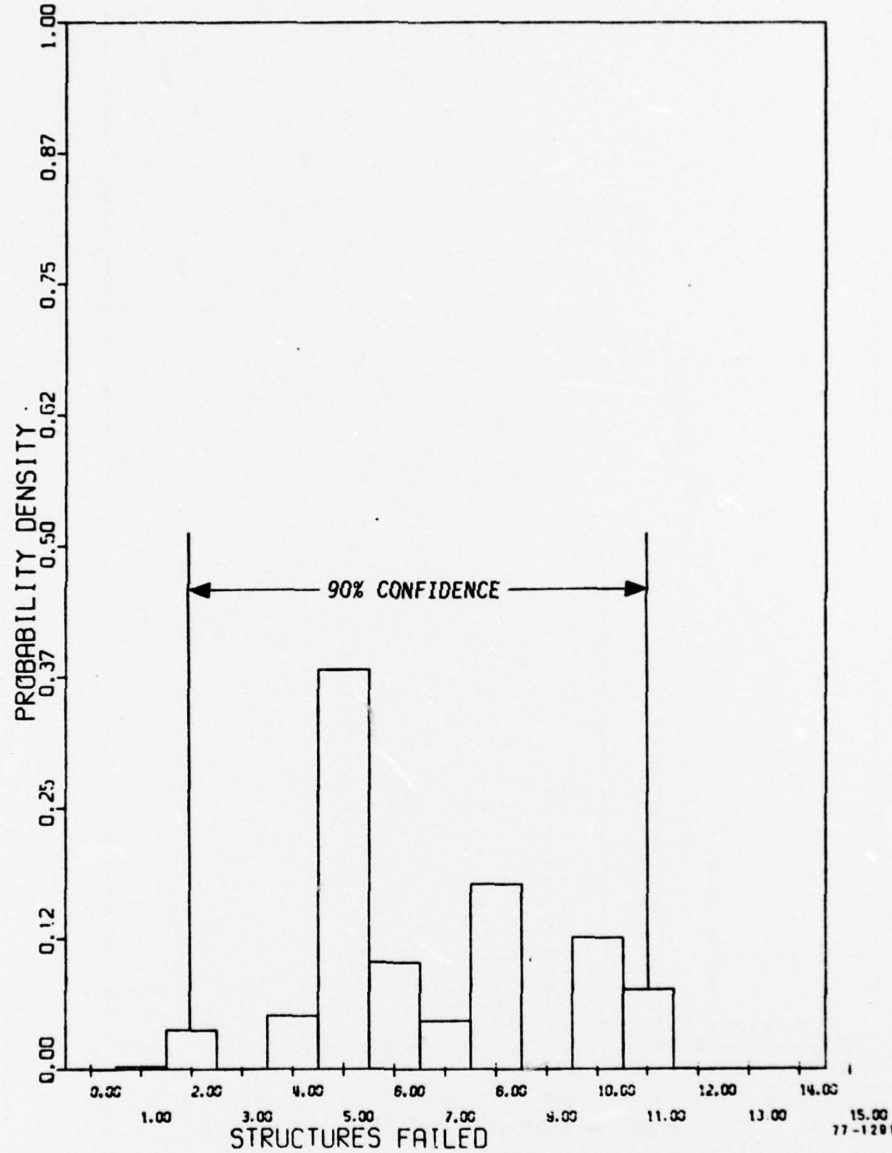

Figure 4-6. Fully-Correlated Histogram for Mighty Epic (Factor of 1.1 Uncertainty in Pressure)

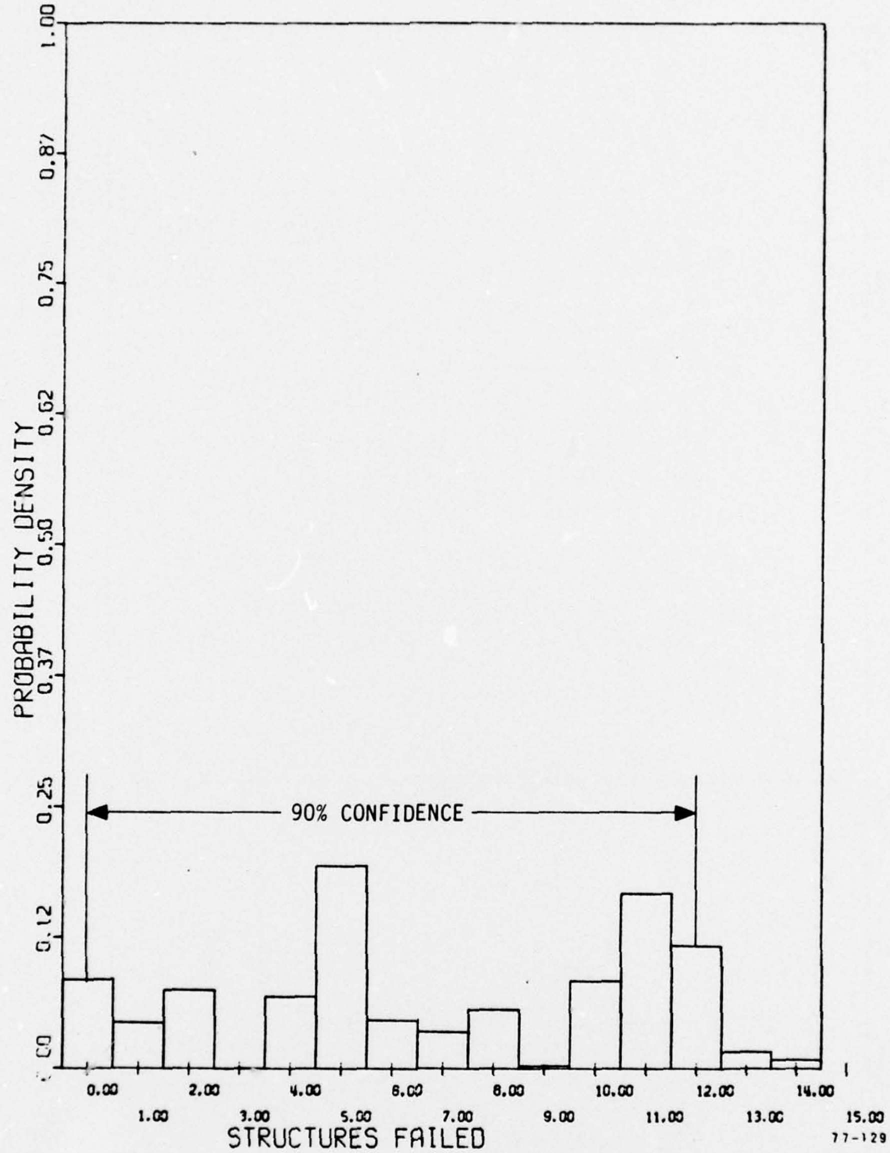

Figure 4-7. Fully-Correlated Histogram for Mighty Epic
(Factor of 2 Uncertainty in Pressure)

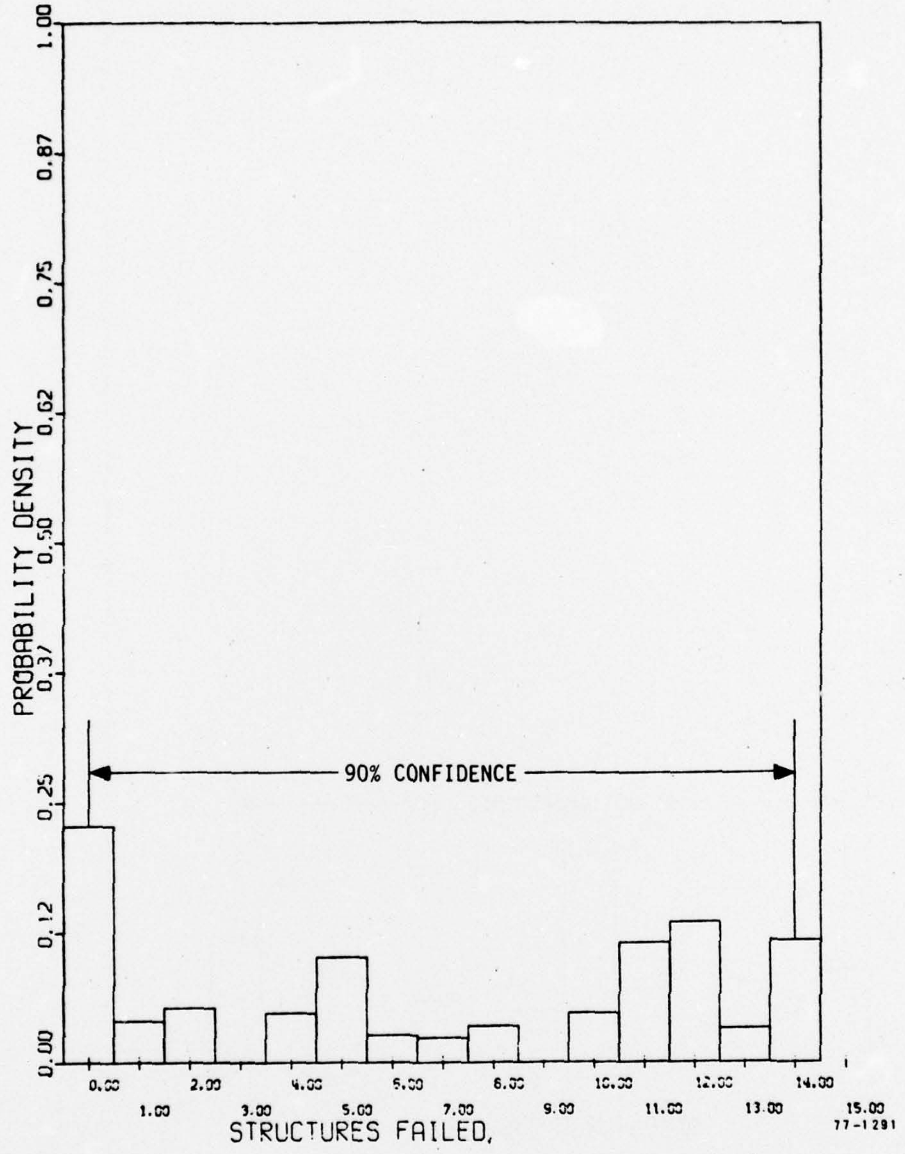

Figure 4-8. Fully-Correlated Histogram for Mighty Epic (Factor of 4 Uncertainty in Pressure)

4.4 DISCUSSION OF CORRELATION

It can be argued that the effects of correlation should be such as to emphasize the extremes of the probability distributions, as was discussed in Section 4.1. In brief, if the properties of the structures are all correlated, then they are more likely to <u>all</u> be over-strength (or all under-strength) and <u>all</u> survive the loading (or all fail). Similarly, if the pressure loading is correlated, then it is more likely that they will <u>all</u> survive (if the pressure is low) or they will <u>all</u> fail (if the pressure is high). Thus, from a physical standpoint, it is understandable why correlation emphasizes the extremes of the distribuitons.

The real situation in the (MIGHTY EPIC/DIABLO HAWK) structures experiment falls somewhere between the "fully-correlated" case (treated herein) and the "fully-uncorrelated" case. For example, the cylindrical structures all were made by the same contractor, which tends to make them highly correlated. However, the rock in which they were placed may vary from one location to another, which tends to make the properties a little less correlated.

Sensitivity studies (Reference 5) have shown that the rock properties (like the ultimate strength, σ_{ult}, and the fracture parameter, k_{sr}) have a significant effect on the structural capability (σ/σ_o) of the tunnels. Since the tunnels are fairly widely separated in the rock, one can argue that these sensitive properties (σ_{ult} and k_{sr}) are relatively un-correlated, and hence the tunnel strengths should be uncorrelated. Conversely, another sensitive parameter is the concrete strength, f_c' , which is expected to be highly correlated (due to commonality of the manufacturing process). Thus, there are possible arguments for high correlation among the test structures. It seems clear at this point that the actual structures in (MIGHTY EPIC/DIABLO HAWK) fall somewhere between the two extremes of totally uncorrelated and fully correlated.

5. PROBABILITY CALCULATIONS FOR SPHERES

In addition to the lined tunnels in rock (which were designed by Merritt CASES), the (MIGHTY EPIC/DIABLO HAWK) tests include hollow spherical concrete structures (designed by Agbabian Associates). Reference 6 discusses the rationale behind the test design for the concrete spheres, and Reference 7 presents the equations used for designing the individual spheres themselves. It is noteworthy that several fundamental differences exist between the cylinder design (which was developed by Newmark, Reference 8) and the sphere design (which is based on Haynes's work, Reference 9).

For example, Newmark's tunnel design procedure involves using a steel liner and allows plastic deformation of the steel, the surrounding concrete, and the grout/rock medium. Conversely, the sphere design uses no steel liner, involves fiber-reinforced concrete and assumes elastic behavior of the concrete and rock. The tunnel design by Merritt CASES (Reference 5) accounted for uncertainties by using relatively standard design practice and considered maximum and minimum values of rock properties, concrete properties, etc. On the other hand, Agbabian Associates used a probabilistic design procedure (based upon a paper by Ang and Cornell, Reference 10) to account for uncertainties in material properties, etc. Thus, it is clear that many basic differences exist between the cylinder and sphere design procedures. These differences are worth examining to increase our understanding of the design of deep-based structures.

Other questions arise with respect to the sphere design which concern probabilistic design in general. The results presented herein and in Reference 1 are based on a Monte Carlo* technique, which is more general and can treat wider variations in the data than the method of Ang and Cornell (Reference 10). The latter approach is based on a perturbation procedure which contains the assumption that

$$\frac{\sigma_i}{\mu_i} << 1 \qquad (5\text{-}1)$$

where μ_i is the mean value of the $i^{\underline{th}}$ (uncertain) parameter (e.g., Young's modulus for rock, etc.)

and σ_i is the standard deviation of the $i^{\underline{th}}$ parameter.

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*For a discussion of the term "Monte Carlo" technique, see Reference 11.

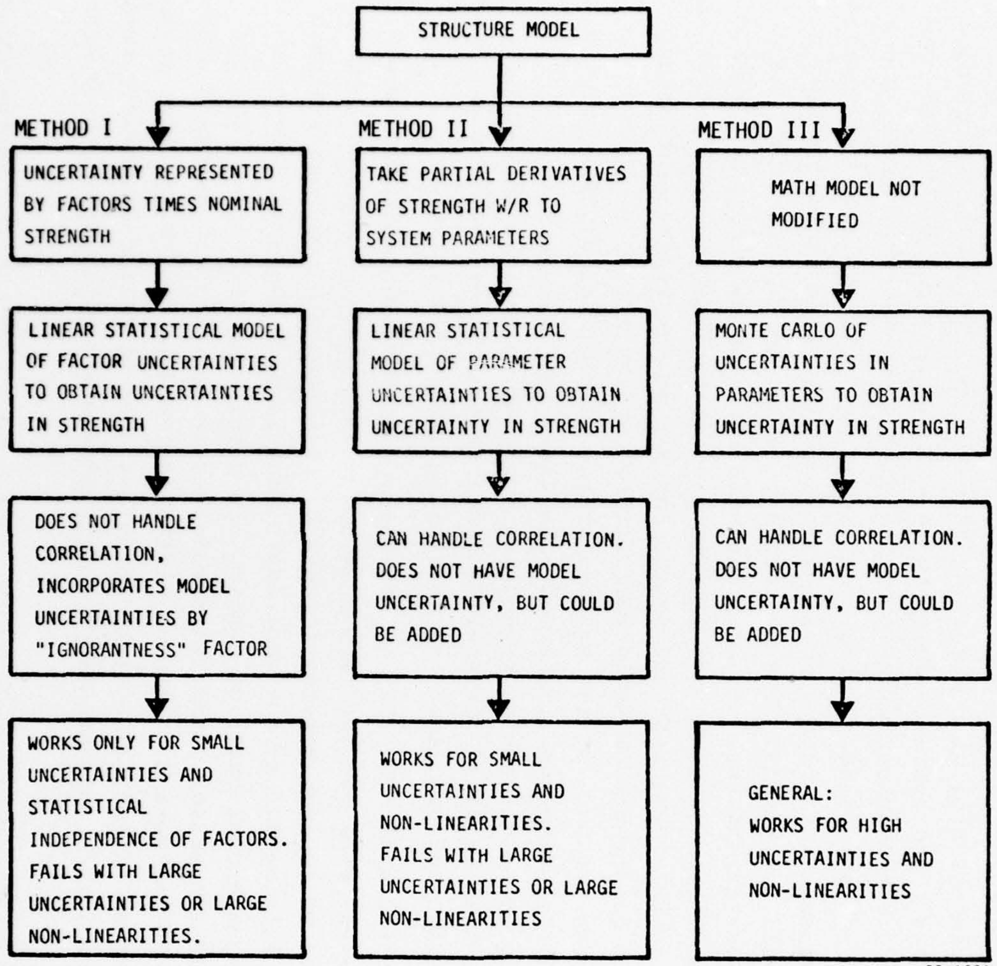

Figure 5-1. Comparison of Three Approaches to Computing Structural Uncertainty

Rather than go into a theoretical comparison of the probabilistic methods used by the various contractors, it is probably of more value to describe the individual approaches and show their strengths and shortcomings. Figure 5-1 shows three methodologies which have been used to evaluate the effects of uncertainties in structures on structure strength. The first method, I, (Reference 7), uses a series of factors multiplied times the nominal strength to express the variation in the actual strength. The assumption is made that the uncertainty in these factors is small compared to the mean. The assumption is also made that there is no statistical correlation between factors. The approach does handle systematic error by the incorporation of an ignorance uncertainty. The approach works only for small uncertainties and if large uncertainties are introduced, will produce very biased results. Consequently, this approach should be used with care, and always with very small variations. The methodology is not applicable to problems with large non-linearities. A second method, II, used by Merchant (Reference 12) uses a perturbation technique wherein partial derivatives of strength are taken with respect to each of the system parameters. A linear statistical model is then constructed wherein the variance of the strength becomes a function of the products of the squares of the partial derivatives and the variances of the parameters. The model as used did not contain correlation, but mathematically is capable of handling correlation between the parameters. Again, this model is valid for small uncertainties and very small non-linearities, but will fail with large uncertainties and non-linearities.

The third appraoch, III, is basically a Monte Carlo method, wherein the model is always preserved, and a simulation is used wherein sampling from the uncertainties of the parameters produces a distribution of strengths of the structure. This method is the most exact, but is also the most expensive when the computational costs of running the simulation are high. From the standpoint of comparison, this third model is usually used in checking other models such as the Method II linear statistical model. For very small perturbations, Methods II and III agree. There has been no direct comparison between Methods I and III in this problem to substantiate the final validity of Method I.

Although the Monte Carlo technique can be applied to the sphere design, and fragility curves can be derived for spheres, no move was made to develop an attendant computer program. The sphere problem was studied sufficiently to outline the flow of the calculations, however, and the results are given in Appendix A. Another problem which was briefly investigated concerns the development of elastic-plastic design of spheres, which is discussed in the paragraphs which follow.

The Newmark procedure (described in Reference 8) allows for elastic-plastic deformation of the concrete liner and the rock. Newmark assumed that the tunnel was very long (the plane strain approximation) and axi-symmetric (no variation in the circumferential direction, θ). Consequently, the tunnel problem is taken to be <u>one-dimensional</u> - i.e., variations along a radius, r, only are allowed. Similar approximations (e.g., that for a sphere the loading is spherically-symmetric) allow the sphere problem to be treated as one-dimensional (variations allowed in the radial direction only). Thus the question of whether or not one can develop a "spherically-symmetric Newmark procedure" for spheres in rock naturally arose.

This problem, of a spherical cavity in an infinite elasto-plastic medium, was recently analyzed by Durban and Baruch (Reference 13). Reference 13 is based on Durban's DSc thesis, and the approach is not as simplified as Newmark's procedure. However, Durban presented a fully non-linear analysis (both material and geometric nonlinearity) which will simplify somewhat if it is limited to small strains (e.g., ε_θ = .01 to .05). Regardless of such simplification, Durban gives solutions in the form of closed integrals which can be readily evaluated numerically. The extension of these results to include a spherical liner in a spherical cavity appears fairly straightforward, providing one is willing to assume perfect bonding (continuity of stress and strain) between the concrete liner and the rock.

Although the subject of a "spherical Newmark" procedure has been examined just briefly, three results stand out:

- The spherically symmetric problem is somewhat more complicated than the cylindrically symmetric tunnel.
- The solution is amenable to numerical integration and is expected to be fairly simple to implement on the computer.
- It is not obvious that the added complexity of allowing elastic-plastic behavior would result in an improved design (relative to the semi-empirical procedures which Haynes has developed for spheres; see Reference 9).

Further work to answer questions about the elasto-plastic design of spheres in rock was beyond the scope of this report.

6. CONCLUDING REMARKS

A primary conslusion of this study is that 5 or 6 cylindrical structures are expected to fail in the DIABLO HAWK test, with 5 tunnels being heavily damaged. The effects of initial ovaling (i.e. initial imperfections) from the MIGHTY EPIC test are thought to be insignificant, however this estimate remains to be proven. To adjust the DIABLO HAWK test and improve the probability of obtaining better design data, it is tentatively recommended that more new cylinders be added to the test.

The effects of correlation of the structural strengths and of the applied pressure loading were examined, and it was found that correlation emphasizes the extremes of the probability distrubutions. That is, the probability of a few failures increases, and the probability of many failures increases. The result is that the confidence bounds of the expected number of failures increases. For example, without correlation, 5 to 6 failures are predicted. For DIABLO HAWK, whereas with full correlation from 3 to 8 failures are predicted.* The actual physical problem will lie somewhere between these two extremes of totally uncorrelated and fully correlated.

The study showed it is a relatively straightforward problem to develop fragility curves for spheres, but time and money constraints would not allow their development herein. A "spherical Newmark" procedure, for elasto-plastic design of spheres in rock, might also be developed, but it is not expected to be as simple as the cylindrical tunnel procedure.

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*These predictions assume a factor of 1.1 uncertainty in the free-field pressure.

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APPENDIX A

FLOW-CHART FOR SPHERE CALCULATIONS

Agbabian's calculations (Reference 7) were reviewed to determine how fragility curves for spheres could be produced. A possible means of generating the fragility curves is shown in Figure A-1.

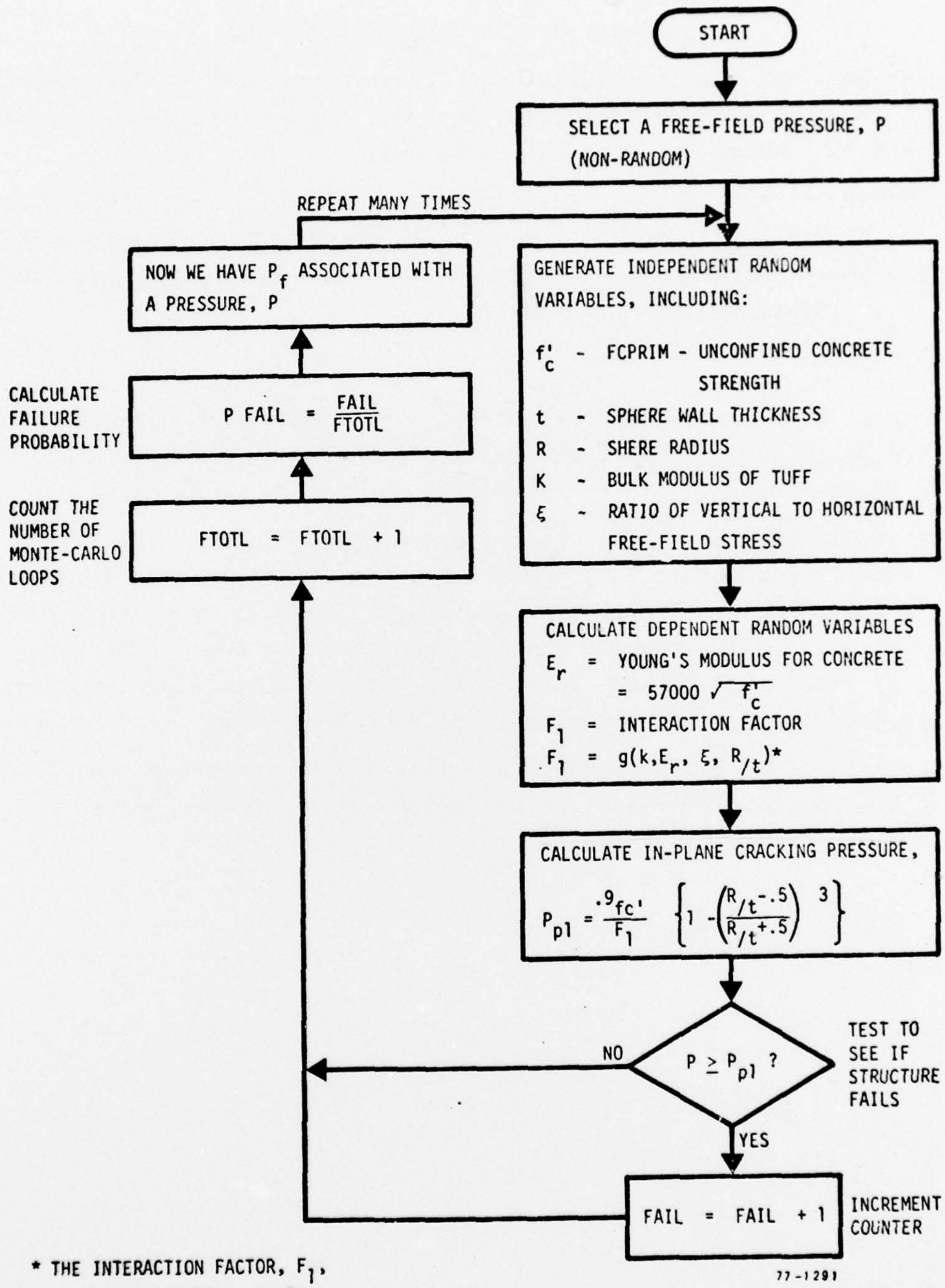
Beginning on the left of Figure A-1, one selects a free-field pressure, P. Then, by generating independent random variables and using equations which relate these variables, one can calculate a (random) in-plane cracking pressure, P_{pl}. Next, one tests to see if

$$P \geq P_{pl} \tag{A-1}$$

If equation (A-1) is satisfied, a counter is incremented, counting the fact that failure has occurred. Then the failure probability is calculated, and the entire process repeated in a Monte Carlo Loop.

The result is that for a particular pressure, P, there will be generated a corresponding failure probability, P_f. One can thus develop a fragility curve, $P_f(P)$, of failure probability as a function of free-field pressure. It might be worthwhile to simultaneously use another failure criteria (say implosion) in addition to in-plane cracking. Then two fragility curves (one for each failure criterion) would be developed. (See Figure A-2).

Correlation among the random properties and the pressure loading could also be treated, following the guidelines given in the body of the report.



* THE INTERACTION FACTOR, F_1, IS A FUNCTION OF K, ξ, E_r AND R/t. SEE REFERENCE 7

Figure A-1. Flow Chart for Generating Fragility Curves for Spheres

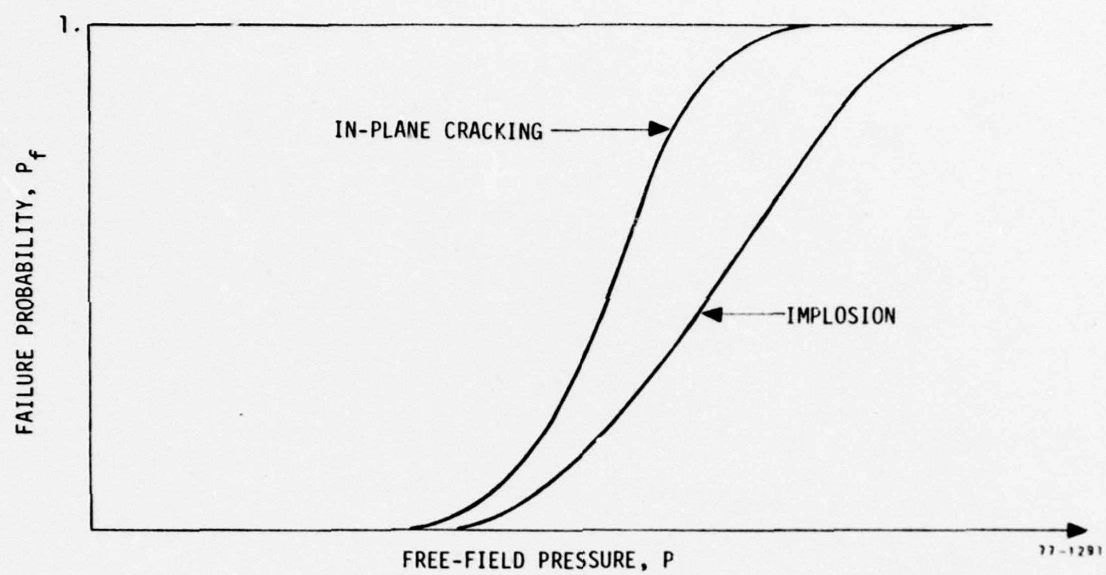


Figure A-2. Sketch of Anticipated Fragility Curves

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